NUMERICAL MODEL FOR MOVABLE BED AS A TOOL FOR THE SIMULATION OF THE RIVER EROSION A CASE STUDY

Solichin [1]

***Abstract**: A serious erosion problem takes place in Cipamingkis River in west Java, Indonesia. As a consequence, river protection works are frequently damaged. The process is probably caused by sand and gravel dredging activities in the river downstream. This problem has not been treated until now. In an attempt to find an appropriate solution, a numerical model to simulate th interaction between bed load transport and bed stability has been developed. At first, the fixed bed model was developed and calibrated using measured water level data from the Rhein river in Germany. By means of this model, a movable bed model has then been developed. The rate of the bed load transport in Cipamingkis river was calculated using Meyer-Peter and Müller formula. For this, the monthly average flood event was chosen as boundary conditions. In order to forecast the effects of existing weir and dredging, the movable bed model has been applied for three different conditions: river without weir and dredging, river with weir and without dredging, an river with weir and dredging. The result shows that the developed model is capable to predict the interaction between bed load transport and bed stability.*

***Keyword:** numerical movable bed model, bed load transport, dredging, erosion.*

Introduction

From the history of civilisation, humankind has tended to concentrate its activity in river basins. This has led naturally to the imposition of man-made changes on rivers, through a water resources management activities: for example, through construction and operation of dams and reservoir for flood control, hydropower generation, navigation, waste disposal in rivers, weirs for irrigation, etc. (Lee 1997). However, such interventions often create new problems, possibly diminishing the utility of river as a resource and often negating the beneficial effects associated with the original plan.

Besides the above mention activities, rivers have other function to produce sand, gravel ang boulders which are dredged again for construction materials. Dredging activities, if not regulated, can damage river natural function and consequently may create environmental pressure. Many rivers in Indonesian at the moment are in critical situation as a consequence of intensive and extensive dredging activities. The following examples can highlight the problem: local scouring around the bridge piers, sliding of dikes due to degradation of channel bed, and during dry season intake structures lies high and dry because of the dropping of river bed level.

This study is focused on simulation of the flow and sediment transport in order to predict the interactions between the weir construction and the dredging activities. With regard to influence of the erosion, a new model has been developed. The model is also applicable to both steady and unsteady conditions. It has been applied to simulate the bed evolution of Cipamingkis River in Indonesian, where a weir was built in 1983, to demonstrate its practical applicability. One year after the construction of weir, extensive dredging activities were started. As result, the initial slope (about 1 %) of the Cipamingkis River was increased by 0.13 %. That means, the bed of river was

[1] Staff member of Hydraulic Laboratory, Department of Civil Engineering, Sebelas Maret University, Solo, Indonesia.

deepening by 4.43 m.

Figure 1: Cipamingkis Weir

Basic Equation

Equation for Hydraulic Routing

The unsteady one-dimensional open channel flow equations can be derived from the principles of conservation of mass and momentum. The resulting equations are hyperbolic, non linear, firs-order partial differential equation, known as the de Saint Venant equation (Chow 1959; Cunge 1980). They can be expressed in terms of discharge and water level (other combinations such as discharge-depth, velocity-water level, etc., are also possible) as the dependent unknown variables.

The following two laws form the theoretical basic for hydraulic routing:

The law of conservation of mass

$$\frac{\partial y}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = 0\,, \qquad (1a)$$

and the momentum equation:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} + gA\frac{Q|Q|}{K^2} = 0\,, \qquad (1b)$$

where x and t = independent variable representing space and time, respectively; $Q(x,t)$ = discharge; $y(x,t)$ = water level; $b(x,t)$ = river width at that water level; $A(x,t)$ = cross sectional area; and $K(y)$ = conveyance.

Eqs. (1a) and (1b) are non linear and do not have an analytical solution. Therefore, they can be solved only numerically, where approximation errors are introduced. The equations can be solved by application of Preissmann's four-point scheme and double sweep method (Solichin 1997).

Equation for Sediment Routing

For sediment routing, the following sediment continuity equation are used (Cunge 1980):

$$\frac{\partial z}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = 0 \qquad (2a)$$

$$\frac{\partial}{\partial x}\left(\frac{Q^2}{2A^2} + gy\right) + g\frac{Q|Q|}{K^2} = 0 \qquad (2b)$$

where z = the level of river bed, G = bed load rate, which is calculated using Meyer-Peter and Müller formula (Solichin 1998). The equation also can be solved by the Preissmann's four-point scheme and then by the double sweep method (Cunge 1980).

Initial and Boundary Conditions

Equations. (1a) and (1b) are hyperbolic differential equations (Abbot 1989). In order to solve it, two boundary condition are needed to close the system and one initial condition is required as starting point.

In sub-critical flows, one boundary condition must be specified at upstream and one-at downstream end. Generally, water discharge as s function of time, $Q(t)$, will serve as an upstream boundary condition, and as downstream boundary condition: either water discharge as a function of time, $Q(t)$, or water stage as a function of time, $y(t)$, can be taken.

In super-critical flows, two boundary conditions, both water discharge as a function of time, $Q(t)$, and water stage as a function of time $y(t)$, must be specified at upstream end. Boundary conditions at the downstream end are not needed since disturbances can not propagate upstream $v > \sqrt{gy}$.

Initial conditions for both sub-critical and super-critical flows must also be specified. Usually the water depth, $y\,(t=1)$ from upstream end to downstream end, can be taken as an initial condition. However, it is assumed

Table 1: Initial and boundary conditions

Boundary Condition	Equation (1a) and (1b)		Equation (2a) and (2b)	
	Upstream end	Downstream end	Upstream end	Downstream end
sub-critical flow	Q(t)	Q(t) or y(t)	Q(t),G(t) and z(t) or y(t)	z(t) or y(t)
Super-critical flow	Q(t) and y(t)	----	Q(t),G(t),z(t) and y(t)	----
Initial condition	Water stage (y) at t = 1 from upstream end to downstre nd			

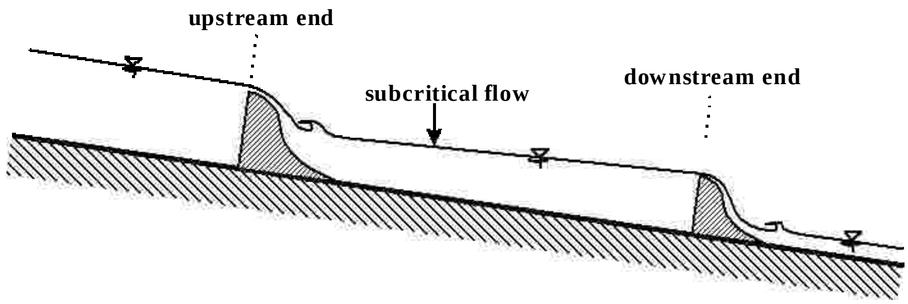

Figure 2: Sub critical flow

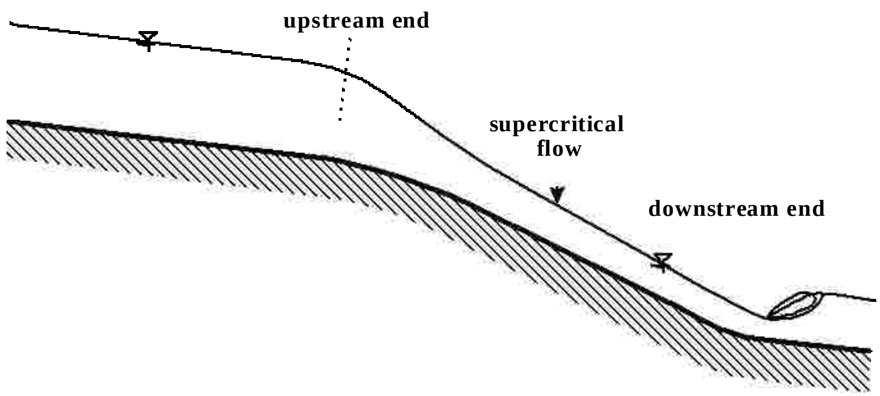

Figure 3: Supercritical flow

that there is no water in the river, but the river has certain previously determined water levels.

In order to solve Equations. (2a) and (2b), two boundary conditions are needed at upstream end, namely water discharge as a function of time, $Q(t)$, and bed load rate as a function of time, $G(t)$. Moreover, the system still needs two boundary conditions both at upstream and downstream ends. This can be achieved by the combination of bottom elevation as a function of time, $z(t)$, and water stage as function of time, $y(t)$.

Table 1, Fig. 2 and Fig. 3 present the above mentioned initial and boundary condition for both equations.

Simulation Model

After solving the differentials equations by the Preissmann's procedures and double sweep method, a simulation program was written on

Fortran 77 codes. In total four programs were developed; they are: 1) program for preparation of the data of cross section geometry (the 'geo-program'), 2) program for preparation of the data of hydrology (the 'hyd-program'), 3) program for calculation of unsteady flow (the 'fix- program'), and 4) program for simulation of sediment transport (the 'mov-program').

Figure 4: Dredging Activity at the River

The simulation procedure is briefly summarized as follows: Due to lack of flow data, the 'fix-program' was calibrated with the data from upper Rhine river between Säckingen und Laufenburg (from 122.144 km. to 129.300 km.). Fig. 5 shows all boundary conditions, Fig. 6 and Fig. 7 show the water stage as the results of calibration.

Based on the results, the 'fix-program' can then be further developed to become 'mov-program'. According to simulated bed load transport, monthly average discharge from Cipamingkis River and bed load rate, are selected as upstream boundary conditions, which is shown in Fig. 8

Fig. 9 shows the water stage as downstream boundary condition. At the upstream end, the bed elevation is assumed constant. As first the river without weir and dredging activities was simulated, and the river with weir at upstream end (that means, without additional bed load transport from upstream) was simulated. Finally, the river with weir at upstream end and dredging activities are simulated. The result is shown in Fig. 10.

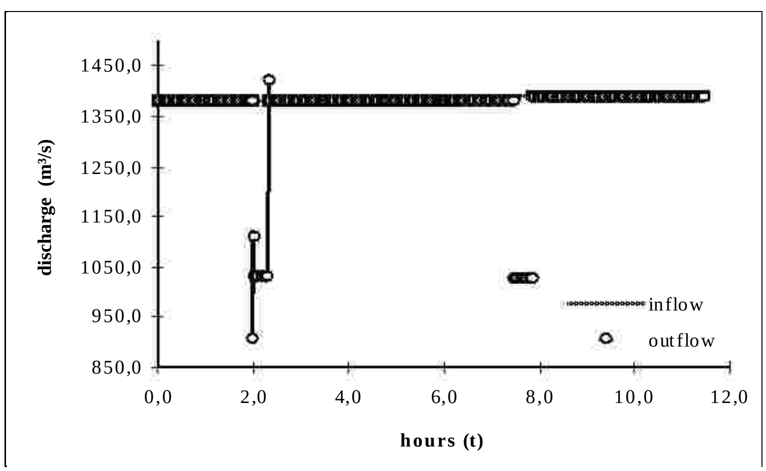

Figure 5. Discharge of Rhine River

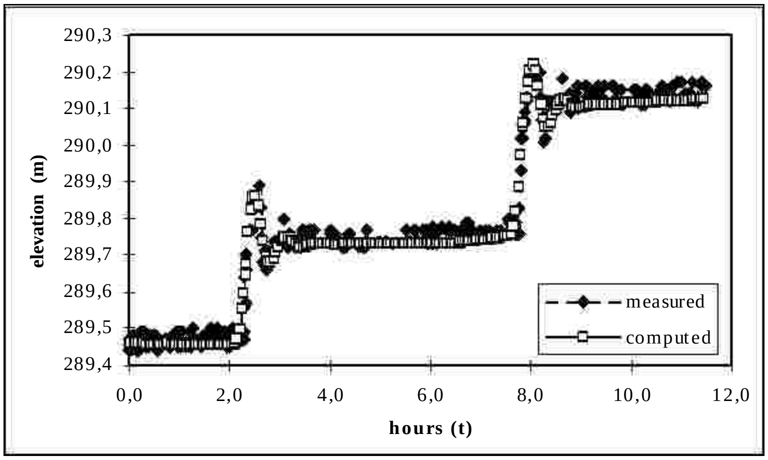

Figure 6. Water stage at sta. 122.562

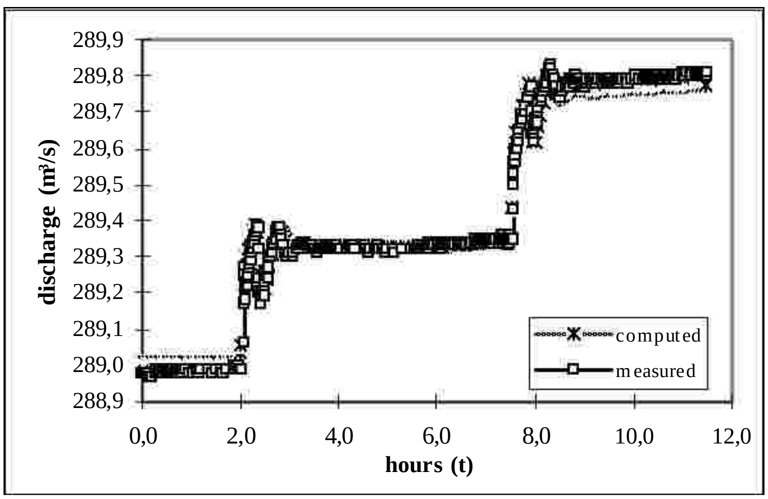

Figure 7. Water stage at sta. 128.092

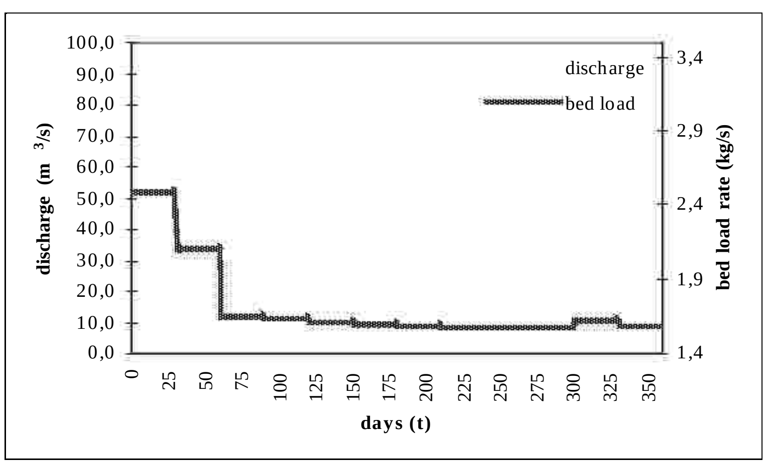

Figure 8. Monthly average discharge and bed load rate

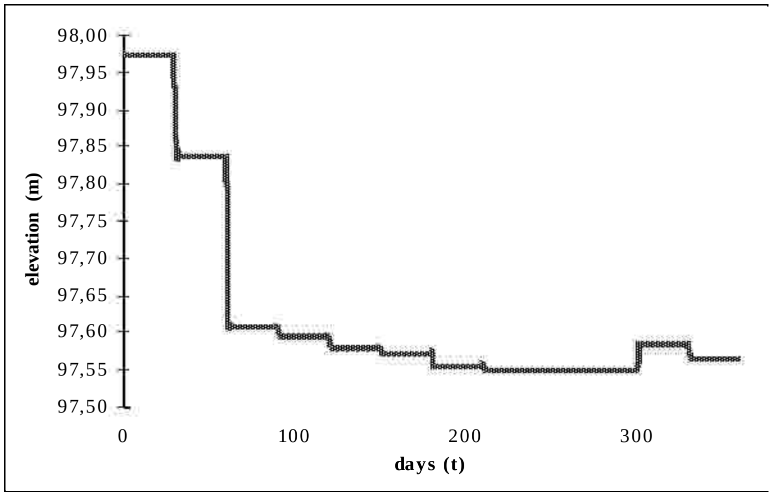

Figure 9. Downstream boundary condition

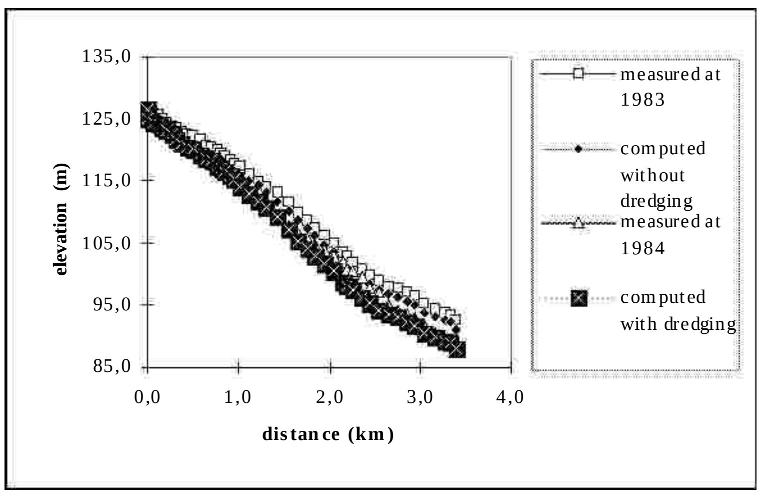

Figure 10. Bed evolution

Result and Discussion

The simulation of bed erosion without dredging shows that even one year after the construction of weir, the slop of the river bed has increased only by 0.04 %. This means that the river bed will be deepen by only 1.36 m. However, the simulation with dredging activities shows that the slope of the river bed will be increased by 0.13 % one year after the construction of weir, which mean also the river bed will be deepen by 4.43 m. In this case study, the simulated data finely correlate with the measured data (see Fig.10).

Conclusion

This exercise demonstrates the capability of the model to simulate the erosion behaviours of the bed load in the river. The performance of the model has been assessed through a comparison with measured data set from Cipamingkis River.

References

Abbott,M. B., D. R. Basco, (1989). *Computational fluid dynamics an introduction for engineer*. Longman Publishers, Singapore.

Chow, V.T. (1959). *Open channel hydraulics,* McGraw-Hill, New York.

Cunge, J.A., F. M. Holly Jr, and A. Verwey, (1980). *Practical aspects of computational river hydraulics*. Pitman Publishing, London.

Lee, H.Y., H. M. Hsieh, J. C. Yang, (1997). *Quasi-two dimensional simulation of scour and deposition an alluvial channel.* J. Hydr. Engrg., ASCE, July, p. 600-609.

Solichin,(1988). *Berechnung von Sedimenttransportmengen*. Jahresbericht an den DAAD, Bonn.

Solichin,(1997). *Lösung der Saint-Venant-Gleichung als die Grundlage eines hydrodynamisch-numerischen Modells,* Jahresbericht an den DAAD, Bonn.